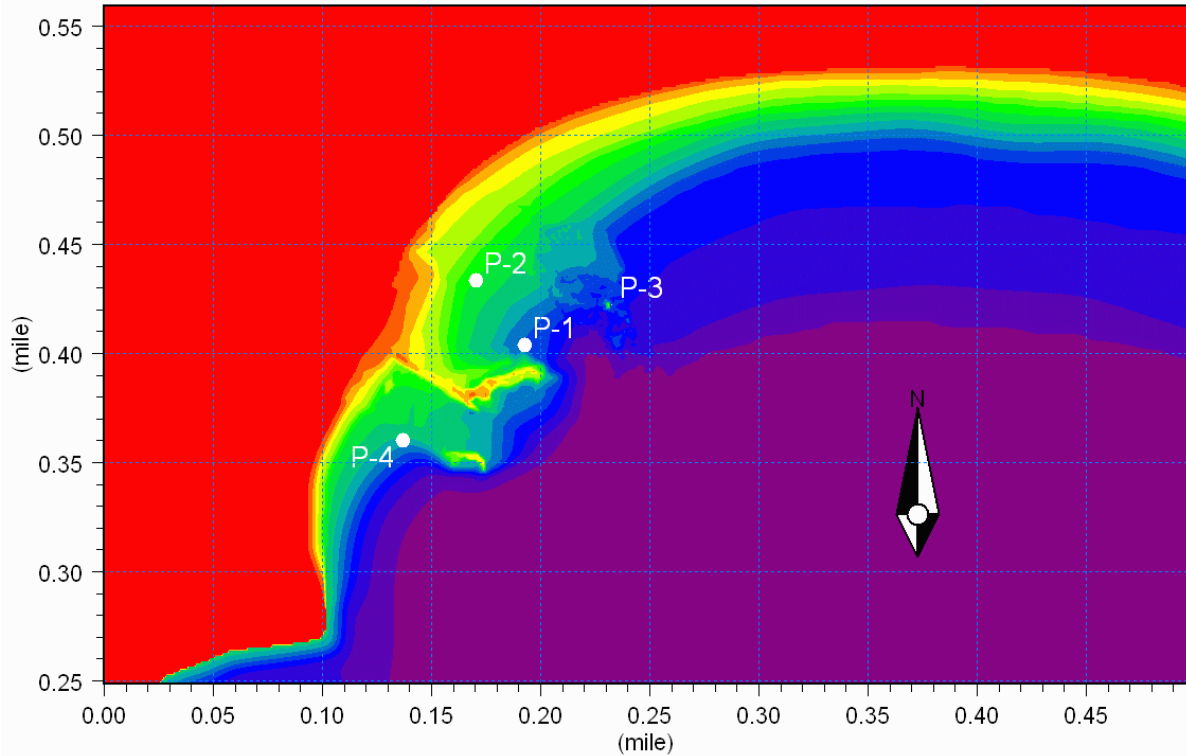


# Shelter Cove Breakwater Rehabilitation Project

## COASTAL ENGINEERING ANALYSES



Prepared for:  
**Humboldt Bay Harbor, Recreation and  
Conservation District**

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# 1. INTRODUCTION

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## 1.1 BACKGROUND

This report presents the results of coastal engineering analyses conducted for the Shelter Cove Breakwater Rehabilitation Project. The breakwater, which was constructed during the late 1970's to protect the boat launch ramp, has deteriorated over the past 25 years. The study was conducted for the Humboldt Bay Harbor, Recreation and Conservation District (HBHRCD), who has been the lead agency for several projects at the Shelter Cove Boat Launching Facility since the late 1970s. The objective of the Breakwater Rehabilitation Project is to make it safer for launching and retrieval of boats from the Shelter Cove Boat Launch Ramp in a manner that surfing and other environmental resources in the vicinity are not impacted.

Moffatt & Nichol had earlier completed an Engineering Feasibility Study for the same project (M&N 2004) for the HBHRCD. Three breakwater improvement alternatives, as listed below, were presented and evaluated at a preliminary level in the feasibility study.

### Alternative 1: Repair In Place

Repair the breakwater in place, raising the crest elevation from approximately 4.5 feet NGVD to 9.5 feet NGVD – similar to the 1979 condition. The rationale for this alternative was to maintain the original breakwater dimensions, thereby providing the same level of protection as before.

### Alternative 2: Repair and Extend

Repair the breakwater in place as for Alternative 1, and extend the outer leg by approximately 80 feet. The objective of this alternative was to enhance the level of protection at the concrete launch ramp, such that it would be more protected than the original construction.

### Alternative 3: Realign Outer Leg

Repair the breakwater as for Alternative 1, but realign the outer leg to be shore parallel. The rationale for this alternative was to increase the size of the protected basin, such that it would provide safe harbor as well as allow launching/retrieval at lower tides than at present, while minimizing any changes to the local wave regime.

The analyses completed for the Feasibility Study concluded that, for the predominant wave conditions considered, Alternatives 1 and 3 would meet the performance objectives of the breakwater rehabilitation (enhance protection from waves while minimizing other environmental impacts). Alternative 1 would bring the breakwater back to as-constructed conditions (ca. 1979), and Alternative 3 would provide similar wave protection while increasing the basin area protected from waves by about 13% (equates to about 10,000 ft<sup>2</sup>). It was also recognized that additional analyses, which would encompass the complete range of wave conditions occurring at the site, would be necessary during design to confirm that environmental impacts are truly negligible.

At a public meeting in Shelter Cove, several boaters expressed interest in having the larger protected basin resulting from Alternative 3 if it did not negatively affect other resources. Subsequently, a Notice of Preparation and Initial Study, with Alternative 3 being the preferred alternative, was prepared by the HBHRCD and circulated for public

and agency review and comment. Comments on the Initial Study were largely related to potential negative impacts on surfing at Wash Rock, potential for bluff and beach erosion in the area, quantification of improvements to navigation, and impacts to the littoral sand system. Many of the comment letters expressed a bias towards Alternative 1 rather than Alternative 3, because of uncertainties associated with a larger footprint of breakwater.

The coastal engineering analysis conducted as part of this study was conducted to specifically address the comments received on the Initial Study.

## **1.2 SCOPE OF WORK**

The following scope of work is included in the report:

- A) Assess Effect Of Breakwater Rehabilitation On Launch Operations
  - 1. Develop a large scale numerical offshore wave model, conduct wave transformation simulations for a range of offshore wave heights, periods and directions to determine wave characteristics jut offshore of Shelter Cove.
  - 2. Use a combination of the above results and the existing wave models developed for the Feasibility Study to simulate local wave characteristics in the vicinity of the breakwater and Wash Rock. Compare results for Alternatives 1 and 3 to Existing Conditions.
  
- B) Assess Effect Of Breakwater Rehabilitation on Surf Conditions At Wash Rock
  - 1. Conduct a hydrographic survey around Wash Rock.
  - 2. Use the existing PMS wave model to simulate wave characteristics (height and direction) in vicinity of Wash Rock for different tidal elevations, incident wave heights, periods and directions. Compare results for Alternatives 1 and 3 to Existing Conditions.
  
- C) Assess Impact On Beach And Bluff Erosion At Pt. Delgada
  - 1. Qualitatively describe morphologic processes in the vicinity of Pt Delgada, based on the wave model simulation results obtained from Task A) and B), other similar studies along the coast, and published scientific principles, experiences and judgment with the objective of comparing Existing Conditions to the proposed Alternatives.
  
- D) Develop Operations & Maintenance Costs
  - 1. Conduct a conceptual analysis of long-shore and cross-shore sediment transport processes, and identify potential sedimentation and erosion areas.
  - 2. Develop a cost estimate for breakwater repair and maintenance dredging.
  
- E) Discuss Construction Logistics and Methods
  - 1. Develop a construction scenario by coordinating with regulatory agencies, potential contractors, District staff and the environmental consultant.
  - 2. Discuss material transportation routes, access issues, equipment type and potential methods of construction.

## 2. BREAKWATER REHABILITATION INFLUENCES

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### 2.1 ANALYSIS APPROACH

For the purpose of assessing effects and performance of the breakwater, it is important to understand the full range of local wave characteristics in the project vicinity and the subsequent effects of the proposed breakwater rehabilitation project. In order to analyze local waves, it is necessary to understand how waves are transformed from offshore to the beach. Since waves can come from many directions, the full direction range of offshore waves needs to be considered. Due to the highly irregular bathymetry off the coast, numerical modeling techniques were applied to analyze the wave transformation from offshore to nearshore regions.

For any numerical model, good boundary conditions (wave height and period) at the edges of the model domain are required. Since offshore wave data are available from the Cape Mendocino Buoy, which is about 20 miles west of Cape Mendocino, the offshore wave model domain has to be extended west to where the buoy is located so that the wave buoy data can be used as boundary conditions for wave model input. However by doing this the offshore wave model domain has to be very large (see Figure 2.3-1), and it is difficult to resolve local coastal features such as Wash Rock which is less than 50 feet wide at mid-tide, and the breakwater which is less than 400 feet in length. One practical method is to divide the wave simulation into two stages as described below, which is what was done for this study :

1. First, simulate wave transformation from the buoy to an intermediate location closer to the shoreline. Simulated wave results then can be extracted at this intermediate location. The intermediate location selected was about ½ mile south of the project site, in water depths of about 30 feet (see Figure 2.3-1);
2. Second, develop a nearshore wave model, which only needs to extend from the intermediate location described above to the shoreline, which can resolve the local coastal features and bathymetry well. The extracted wave results from the first step are used as boundary conditions to drive the nearshore model (see Figure 2.4-1).

### 2.2 OFFSHORE WAVE CONDITIONS

The Coastal Data Information Program (CDIP) includes data from the Cape Mendocino Buoy. A wave rose from about 1 year's worth of data (Mar 1999 to Feb 2000, and Jan 2004 to Feb 2004, see Figure 2.2-1) shows that waves come predominantly from the west through northwest directions (about 81% of the time), with the balance from the south to southwest directions. A wave period rose for the same data is shown on Figure 2.2-2. The maximum recorded wave height was on 10/28/1999 with a significant wave height of 9.42m and a peak wave period of 16.67 seconds from the WNW direction. Average monthly wave heights and periods are summarized in Table 2.2.1. The data show that, on average, wave heights and periods in the summer are smaller than the rest of the year. The significant wave height and peak wave period for the Jun-Aug period is about 1.86m and 9.3 second, while for the rest of the year it is about 2.7m and 11.4 seconds.

The data set does not include extreme wave data during El Nino years, but overall it is considered representative of typical conditions, and can be used for the wave modeling analysis. The Feasibility Study had also included wind data and a wind rose, which is not repeated here.

Table 2.2.1: Average Monthly Wave Height and Period at Cape Mendocino Buoy

Parameter	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Mean Hs (m)	2.76	2.91	3.01	2.60	2.03	2.03	1.95	1.61	2.21	2.58	2.79	3.23
Mean Tp (s)	11.80	12.78	12.59	10.92	9.89	9.79	9.20	8.89	9.91	11.26	11.46	12.11

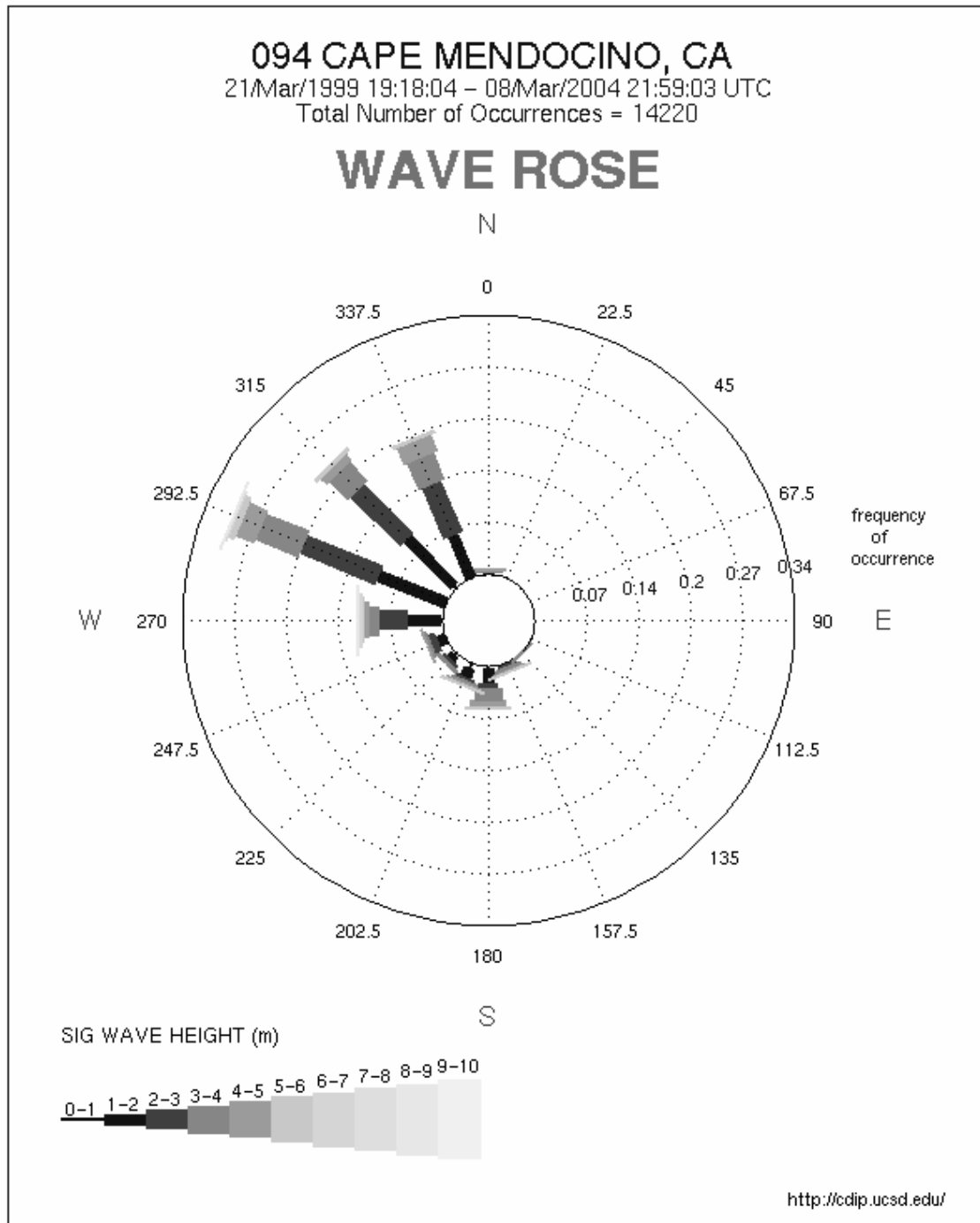


Figure 2.2-1: Wave Height Rose for Cape Mendocino Buoy

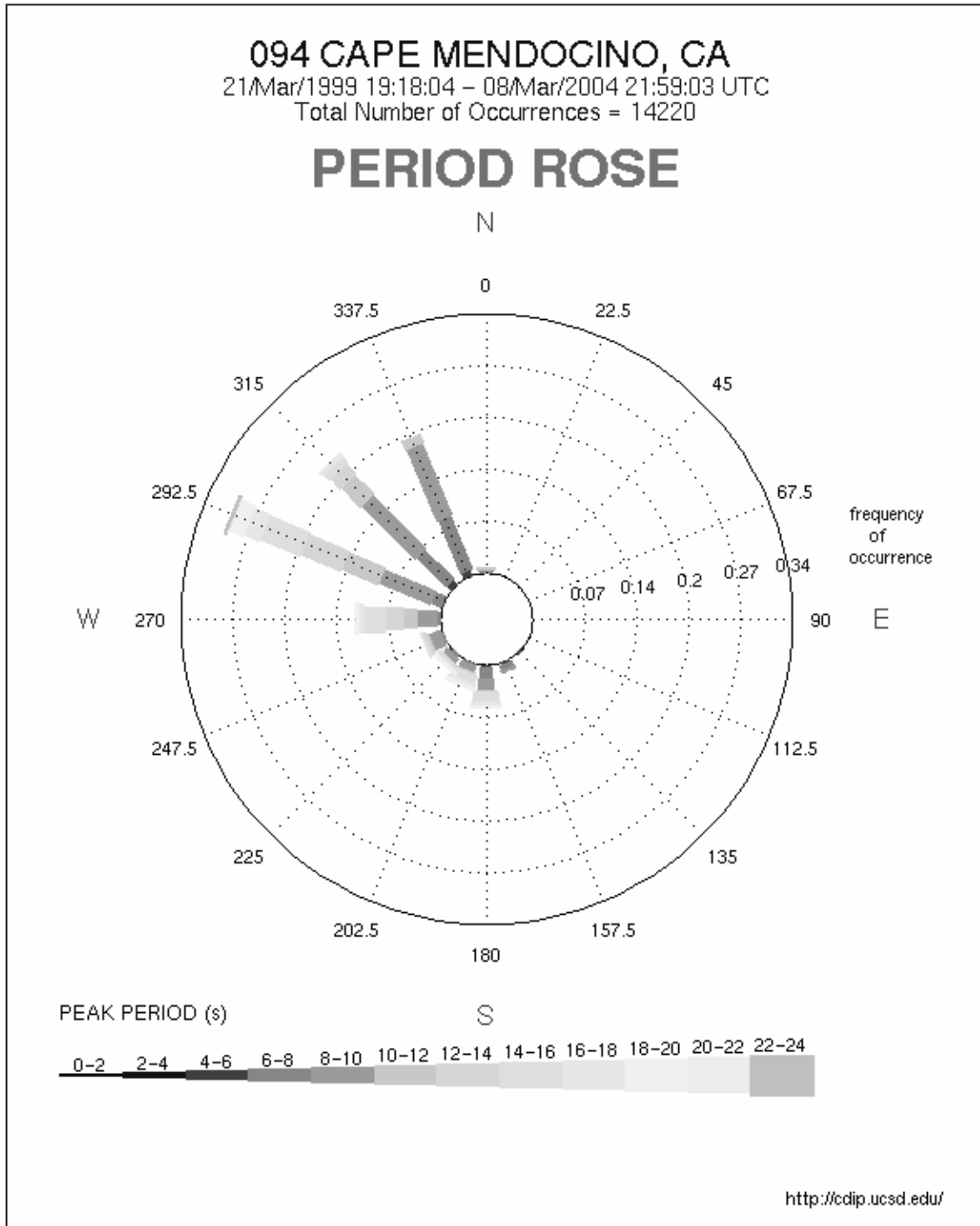


Figure 2.2-2: Wave Period Rose for Cape Mendocino Buoy

### 2.3 NUMERICAL SIMULATION OF OFFSHORE WAVE TRANSFORMATION

To simulate offshore wave transformation, a two dimensional finite difference rectangular grid wave model, MIKE21 NSW, was used. The NSW model is a spectral wave model that describes the propagation, growth and decay of swells and wind waves. The model takes into account the effects of refraction and shoaling due varying water depth, local wind generation and energy dissipation due to bottom friction and wave breaking. NSW is a stationary, directionally decoupled parametric model. It requires input of bathymetry, water surface elevation, incident wave conditions, bottom friction and wave breaking parameters. The basic output from the model is integral wave parameters such as significant wave height, mean wave period and direction, etc.

The model bathymetry file was developed based on NOAA Chart 18620, Pt. Arena to Trinidad Head [Reference 2]. For computational efficiency, a 20-meter grid spacing in the X direction and an 80-meter grid spacing in the Y direction were selected for the offshore wave model.

Like most wave models, the model requires input waves from the model left boundary within a relatively narrow angle band. For incident waves from many directions, separate bathymetry files are usually required. The offshore buoy data show that 99% of waves come from directions between  $180^{\circ}$  and  $337.5^{\circ}$ . Therefore eight wave directions ( $180^{\circ} \sim 337.5^{\circ}$ ) were simulated. For this purpose, three offshore wave model domains were established, as shown in Figure 2.3-1. For waves from  $292.5^{\circ}$ ,  $270^{\circ}$ , and  $247.5^{\circ}$ , model domain 1 was used. For waves from  $337.5^{\circ}$  and  $315^{\circ}$ , model domain 2 was used. For waves from  $180^{\circ}$ ,  $202.5^{\circ}$  and  $225^{\circ}$ , model domain 3 was used.

The wave and period roses indicate that about 70% of the time waves are less than 10 feet high, and about 85% of the time the period is shorter than 15 seconds. Since wave transformation is primarily affected by wave period and depth, 2 wave periods were simulated (12 seconds and 15 seconds). An input wave height is also required in the model, therefore a wave height of 10 feet was used. A tide equivalent to Mean High Water level (+2.1 feet, NGVD) was used in the analysis. A total of 16 offshore wave simulations were therefore conducted to (8 directions x 2 wave periods).

Waves transform in height and direction as they approach the coast, and tend to become more parallel to the bottom contours nearshore. Waves approaching perpendicular to the shoreline (shore normal waves) tend to *shoal* and increase in wave height until they break. Waves approaching at an angle to the shoreline *refract* and reduce in wave height as they change direction to become more shore normal. The amount of shoaling and refraction is also significantly dependent on the period of the waves (long period waves feel the bottom quicker than short period waves). Wave transformation is therefore a function of wave direction, water depth, and wave period.

Figure 2.3-2 and Figure 2.3-3 present simulated wave transformation patterns for all eight incident directions described above. Results are shown for an incident wave height of 9.8 feet and a wave period of 12 seconds (15 second wave periods were simulated, not shown). For presentation purposes, the wave pattern maps in the figures are subsets extracted from the offshore wave model results, and rotated so that true north points upward. Results of the simulations are also summarized in Table 2.3.1. The parameters “*Wave Transformation Coefficient*” (defined as the ratio of the transformed wave height to the incident wave height) and “*Wave Direction at 30-ft*

Contour” (signifying change in direction between offshore and the 30-ft contour) were used to characterize the transformation of waves as they approach the coast.

The results show that waves from the WNW through NNW directions refract substantially as they approach the coast (for example, NNW waves with 15 sec period reduce to 14% of their offshore wave height as they change about 70° in direction), and become more westerly as they approach the coast. Waves from the SW through W directions shoal more than they refract (for example, W waves with 15 sec period increase to 115% of their offshore wave height with a 40° change in direction). Waves from the S through SSW directions do not change appreciably in height or in direction.

In other words, for northwest and south waves, wave refraction dominates over shoaling, while for westerly waves, wave shoaling dominates over refraction. A significant conclusion that can be drawn from the results is that the full direction band of the offshore waves (180° ~ 337.5°) is essentially reduced to a narrow band between 188° and 268° at the 30-ft depth contour. Therefore, for nearshore wave simulation (shallower than 30 feet), it is only necessary to simulate waves from this 188° to 268° direction band to represent the wider offshore wave direction band.

Table 2.3.1: Wave Transformation From Deepwater to 30-ft Depth Contour

Wave Direction (offshore)	Frequency of Occurrence	Wave Period	Transmission Coefficient at 30-ft Contour	Wave Direction at 30-ft Contour
337.5°	18.2%	12s	0.16	268°
		15s	0.14	267°
315°	22.9%	12s	0.31	267°
		15s	0.26	267°
292.5°	30.3%	12s	0.84	238°
		15s	0.86	234°
270°	9.4%	12s	1.05	232°
		15s	1.15	230°
247.5°	3.3%	12s	1.05	228°
		15s	1.20	228°
225°	2.3%	12s	0.98	214°
		15s	1.13	216°
202.5°	5.5%	12s	0.86	199°
		15s	0.91	204°
180°	6.8%	12s	0.85	188°
		15s	0.82	192°
Other	1.3%	-	-	-

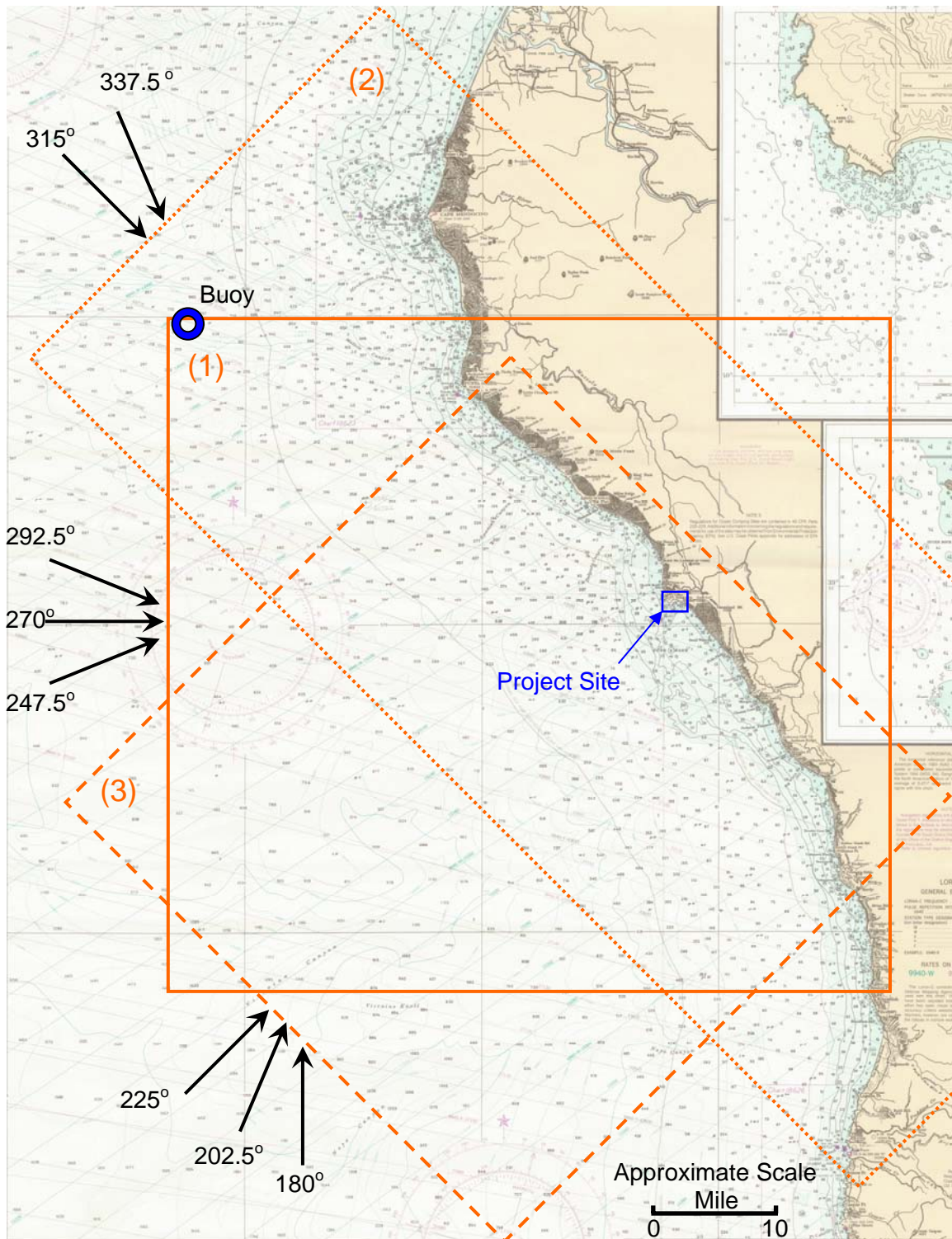


Figure 2.3-1: Offshore Wave Model Domain (NSW Model)

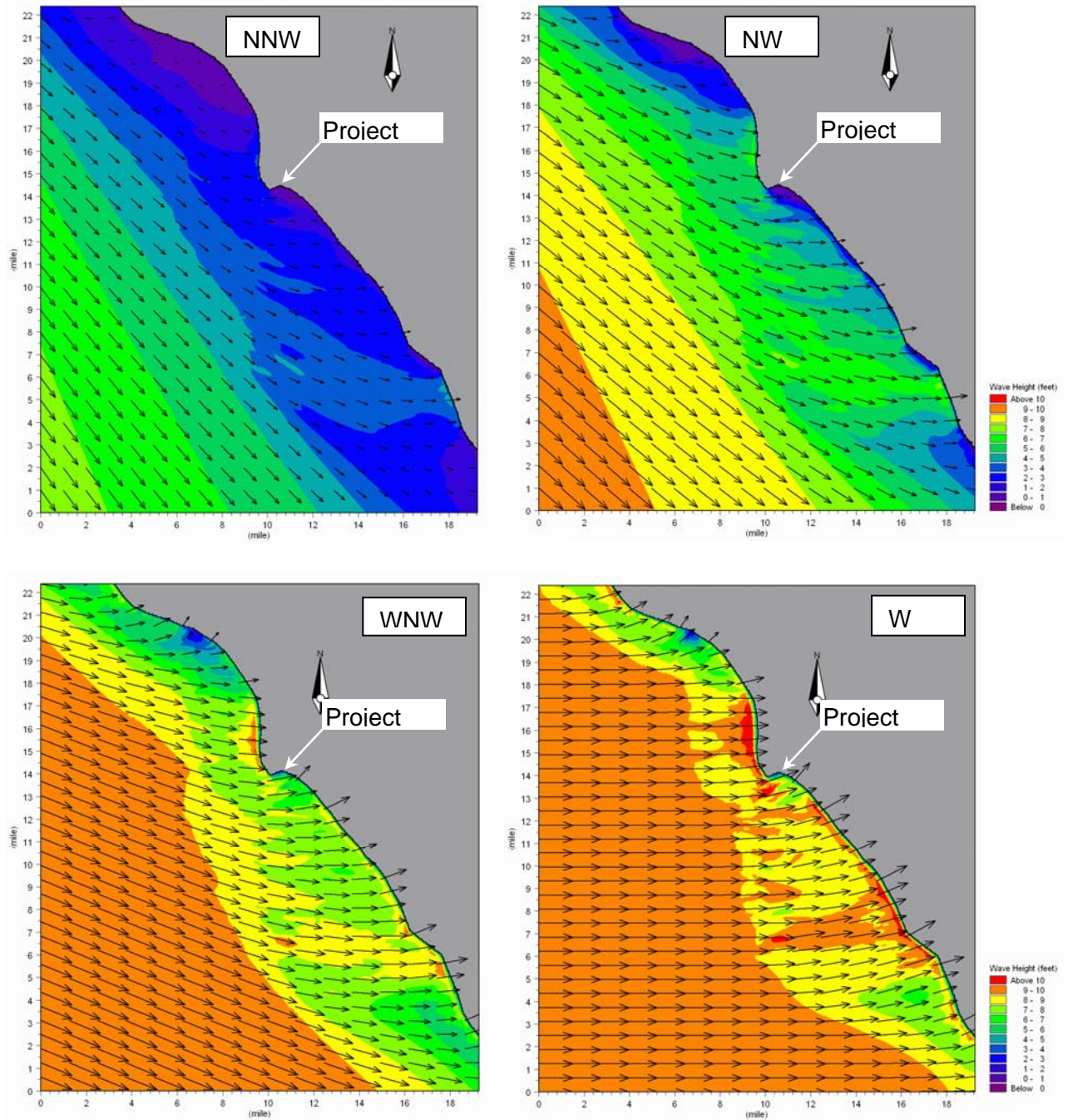


Figure 2.3-2: Wave Transformation For NNW Through W Directions (9.8 ft @ 12 sec)

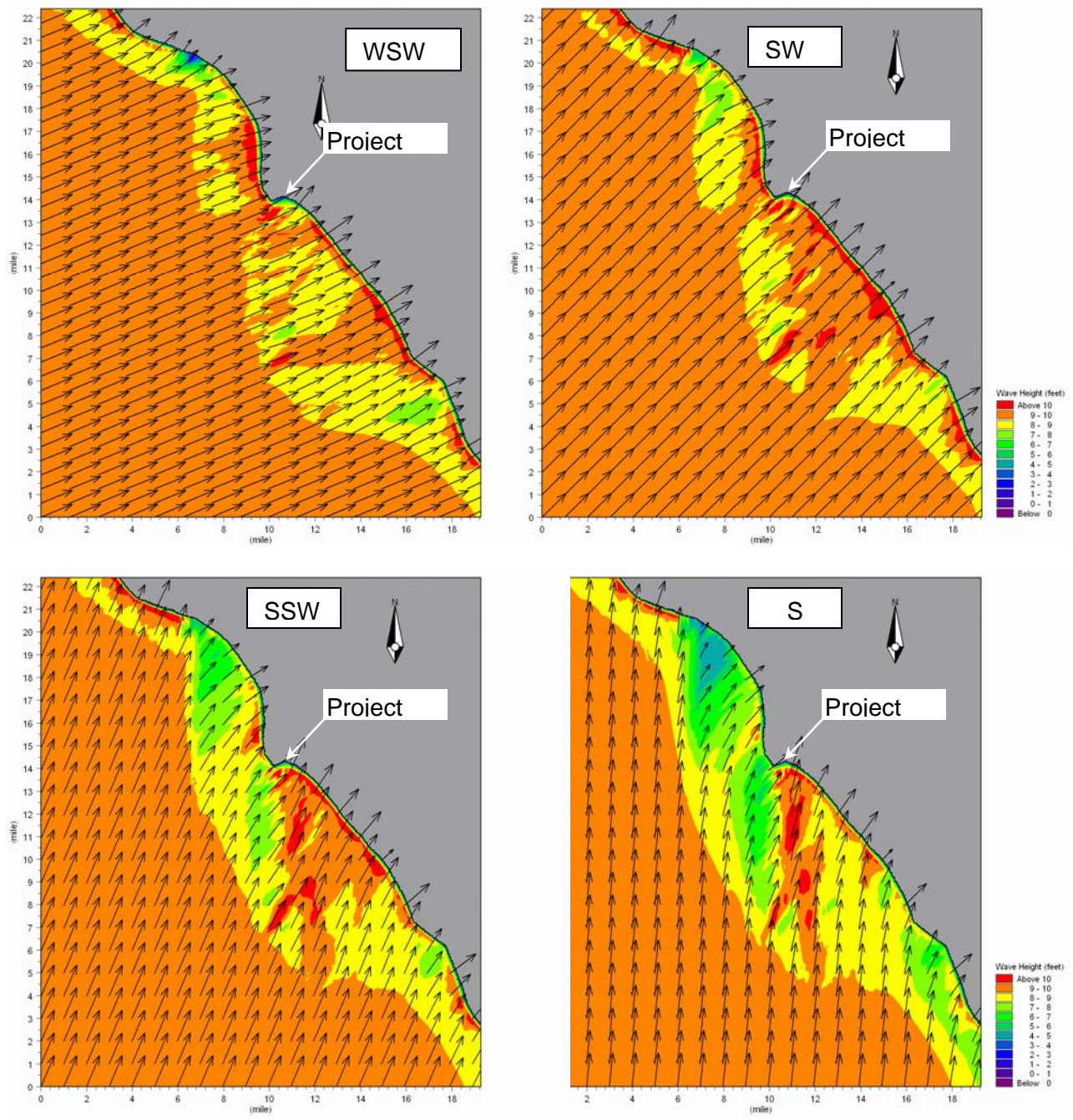


Figure 2.3-3: Wave Transformation For WSW Through S Directions (9.8 ft @ 12 sec)

## 2.4 NUMERICAL SIMULATION OF NEARSHORE WAVE TRANSFORMATION

As shown in the previous section, wave directions at 30-ft water depth are confined to a narrow direction band between 190° and 270°. To simulate waves coming from within these directions, two nearshore wave model domains were constructed. The nearshore domain 1 was used for simulating waves coming from 180°, 200° and 220° directions and the nearshore domain 2 was used for simulating waves coming from 240° and 260° as shown on Figure 2.4-1.

The PMS wave model, originally developed for the Engineering Feasibility Study, was refined to better resolve important local features such as the breakwater and Wash Rock. A 2-meter grid spacing in both X and Y directions was used for the model. The model bathymetry file used for the Engineering Feasibility Study was updated with more recent survey data around Wash Rock obtained from November 2005, which was applied for this study.

The PMS model is a two dimensional rectangular grid finite difference wave model. The model takes into account the effects of refraction and shoaling due to varying depth, diffraction along the perpendicular to the predominant wave direction and energy dissipation due to bottom friction and wave breaking. The basic input data are bathymetry, water level, incident wave condition and various parameters such as bottom friction coefficient and wave breaking parameters. The basic output data from the model are integral wave parameters such as the significant wave height, peak wave period and mean wave direction.

The incident wave boundary conditions for the nearshore wave model were obtained from the offshore wave simulation results described in the previous section, and results from the nearshore wave model were used to analyze the impact of the proposed breakwater rehabilitation project.

Existing conditions, and the two alternatives, Alternatives 1 and 3 [Reference 1] were analyzed in this study.

Four numerical stations were established and simulated wave transformation coefficients and wave directions at these stations were extracted from the model results and compared for both alternatives and existing conditions to evaluate the significance of the breakwater rehabilitation on launch operations, surf conditions near wash rock, and bluff erosion near Pt Delgada. The locations of these stations are shown in Figure 2.4-2 and are described as follows:

### Within Harbor (to assess effects on launch operations)

- Station P-1 - located just on the leeward side of the tip of the breakwater and close to the channel, in a water depth of about 6 feet NGVD. Currently, boats launch at this point at low tide.
- Station P-2 - located near the concrete launch ramp, in a water depth of 2 feet NGVD. Currently, boat launches at this point at high tide.

### Near Wash Rock (to assess effects on surf conditions)

- Station P-3 - located near wash rock in a water depth of about 7 feet NGVD, is used to evaluate the surf conditions in this area.

Near Pt Delgada (to assess effects on bluff erosion)

- Station P-4 - located between Pt Delgada and the breakwater in water depths of about 4 feet NGVD, is used to evaluate the effect of the breakwater on sediment processes west of the breakwater.

Since the nearshore water depth is shallow, wave breaking, in addition to refraction, diffraction and shoaling, will occur. The wave height transformation coefficient will largely dependent on the incident wave condition and water depth, as well as wave direction and breakwater configuration. Therefore, the following wave height/period, direction, water level and breakwater alternative combinations were considered in the simulation.

- Incident Wave Height/Period: 3.3 feet/9 seconds, 7.5 feet/12 seconds.
- Incident Wave Direction: 180°, 200°, 220°, 240°, 260°.
- Water Surface Elevation: Mean High Water (+2.1' NGVD), Mean Low Water (-2.1' NGVD).
- Breakwater Alternatives: Existing Condition, Alternative 1, Alternative 3.

Combining the two incident wave conditions, five directions, two water surface elevations, and three breakwater alternatives, a total of 60 simulations were conducted. Representative results for Existing, Alternative 1, and Alternative 3 conditions are presented on Figure 2.4-3, Figure 2.4-4, and Figure 2.4-5 respectively. The incident wave height and period was 7.5 feet and 12 seconds. The incident wave directions were 180°, 200°, 220° and 240°. MHW was used for water surface elevation. The wave height and direction maps shown in these figures are a subset of the entire nearshore model domain for better presentation in the breakwater vicinity. The figures demonstrate that although incident wave directions were from 180° to 240°, they refract such that the incident waves near the breakwater are from the 170° to 180° band.

It can be seen from the Figures that waves in the leeward side of the breakwater, for both Alternatives 1 and 3, are reduced significantly compared to the incident waves. Only a very small amount of wave energy gets around the tip of the breakwater into the harbor. Results are described in more detail in the next section (Section 2.5).

The shoaling of waves near Wash Rock (station P-3) for westerly waves is also seen on the figures, which demonstrates why waves break here most of the time. No discernible changes in wave height, resulting from breakwater rehabilitation, can be observed near Wash Rock. However, results are described in more detail in the Section 2.6.

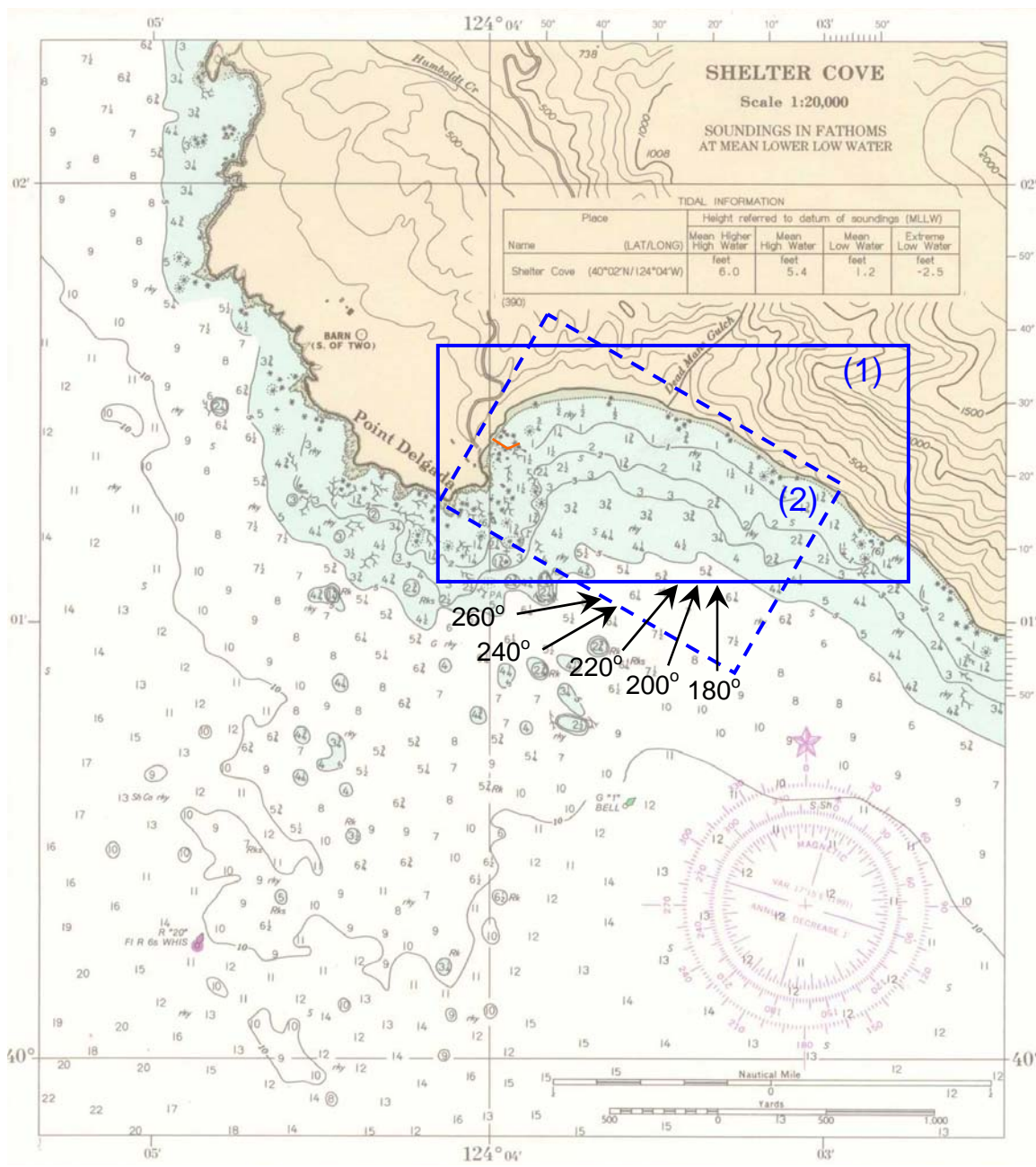


Figure 2.4-1: Nearshore Wave Model Domain (PMS Model)

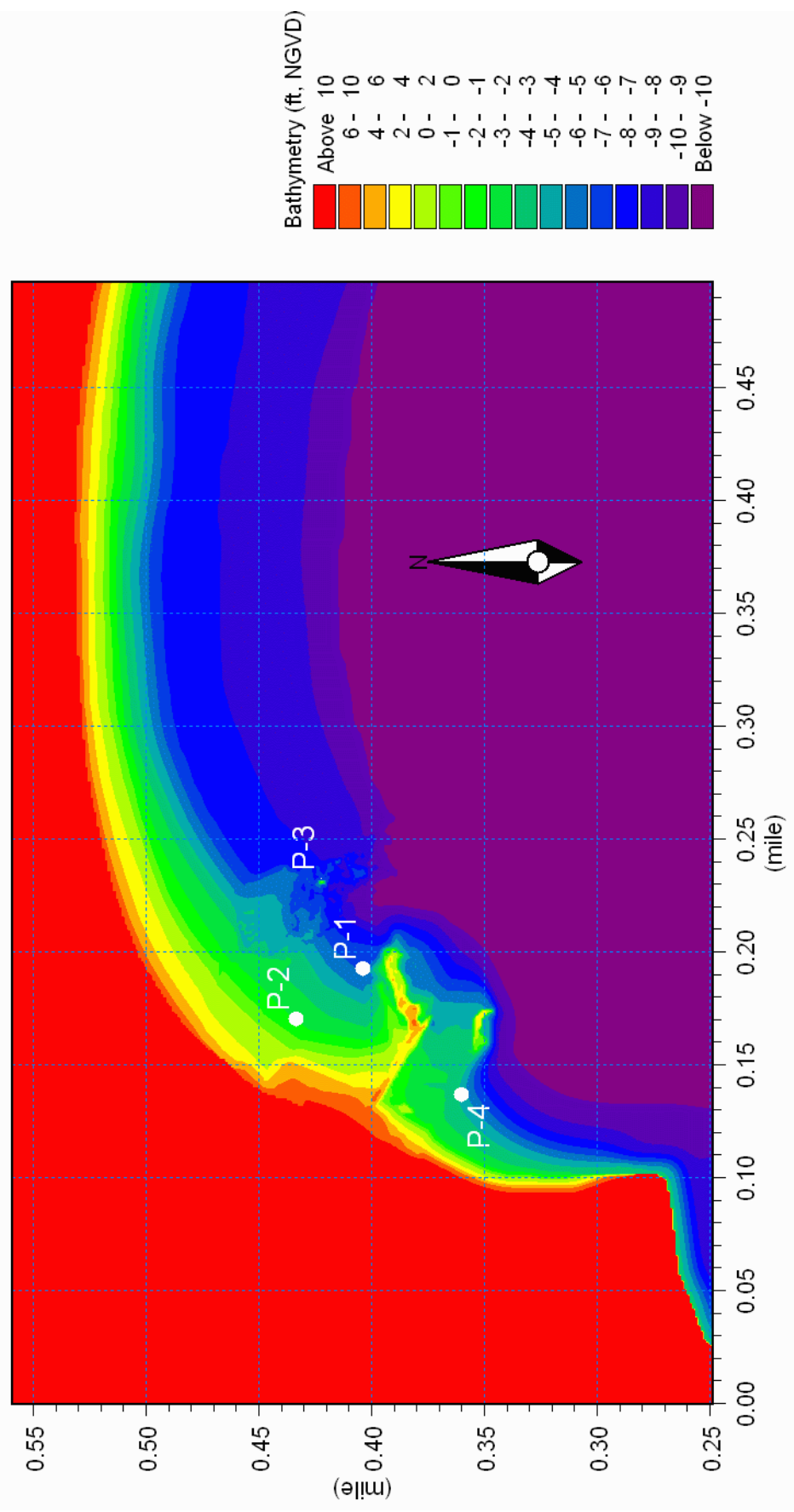


Figure 2.4-2: Model Bathymetry near the Vicinity of the Breakwater

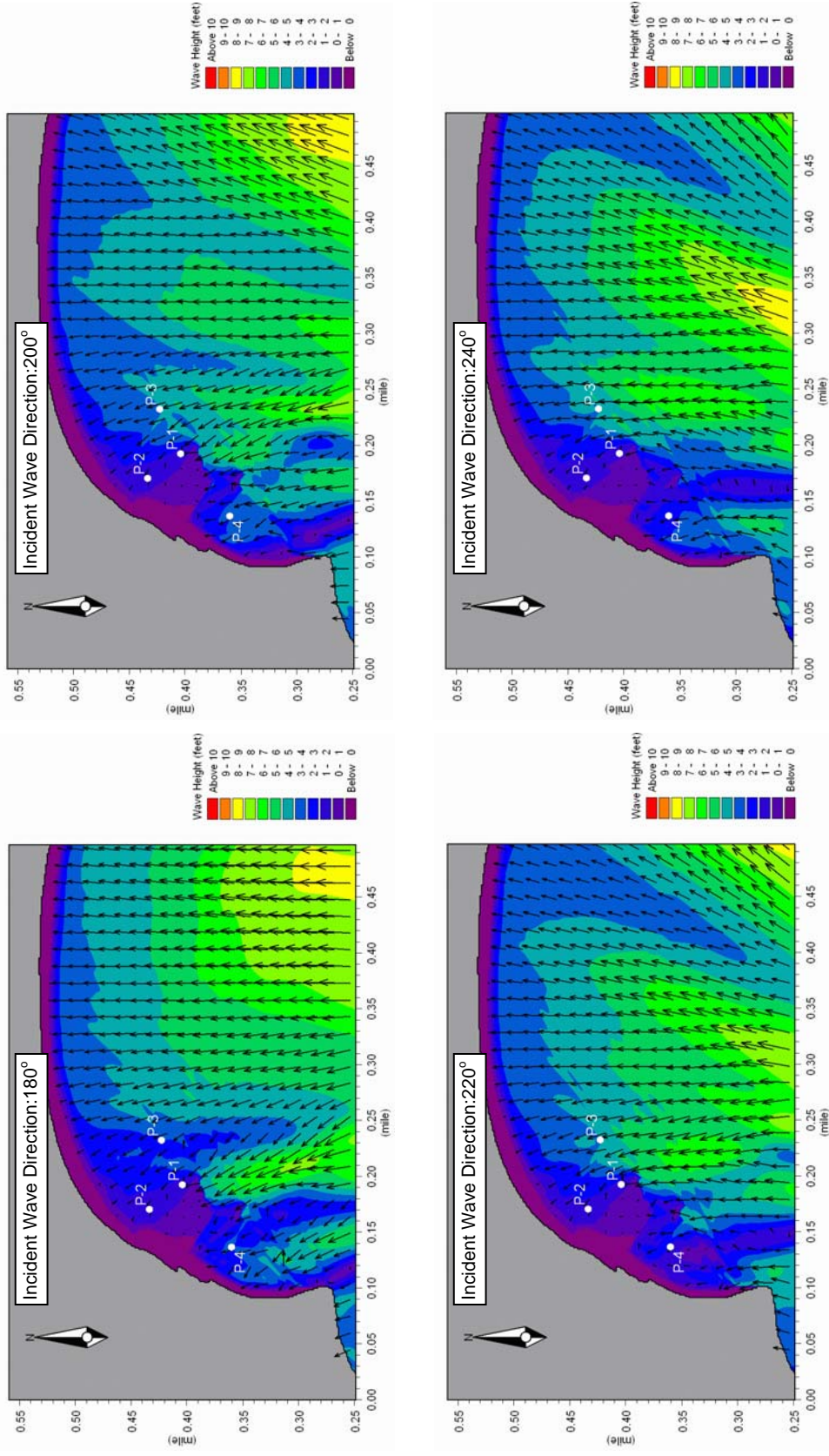


Figure 2.4-3: Nearshore Wave Transformation – Existing Conditions (7.5 ft @ 12 sec, MHW)

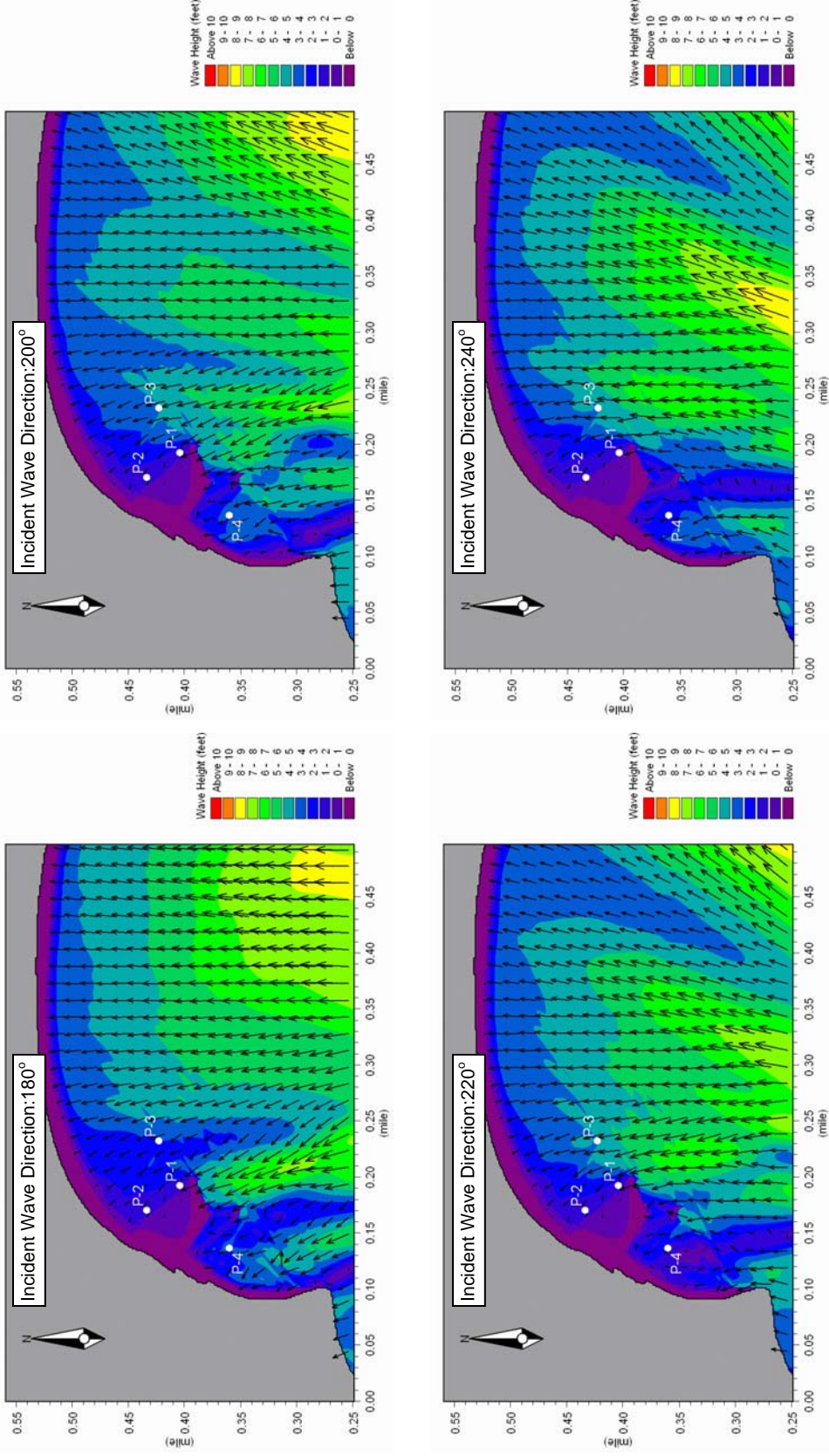


Figure 2.4-4: Nearshore Wave Transformation – Alternative 1 (7.5 ft @ 12 sec, MHW)

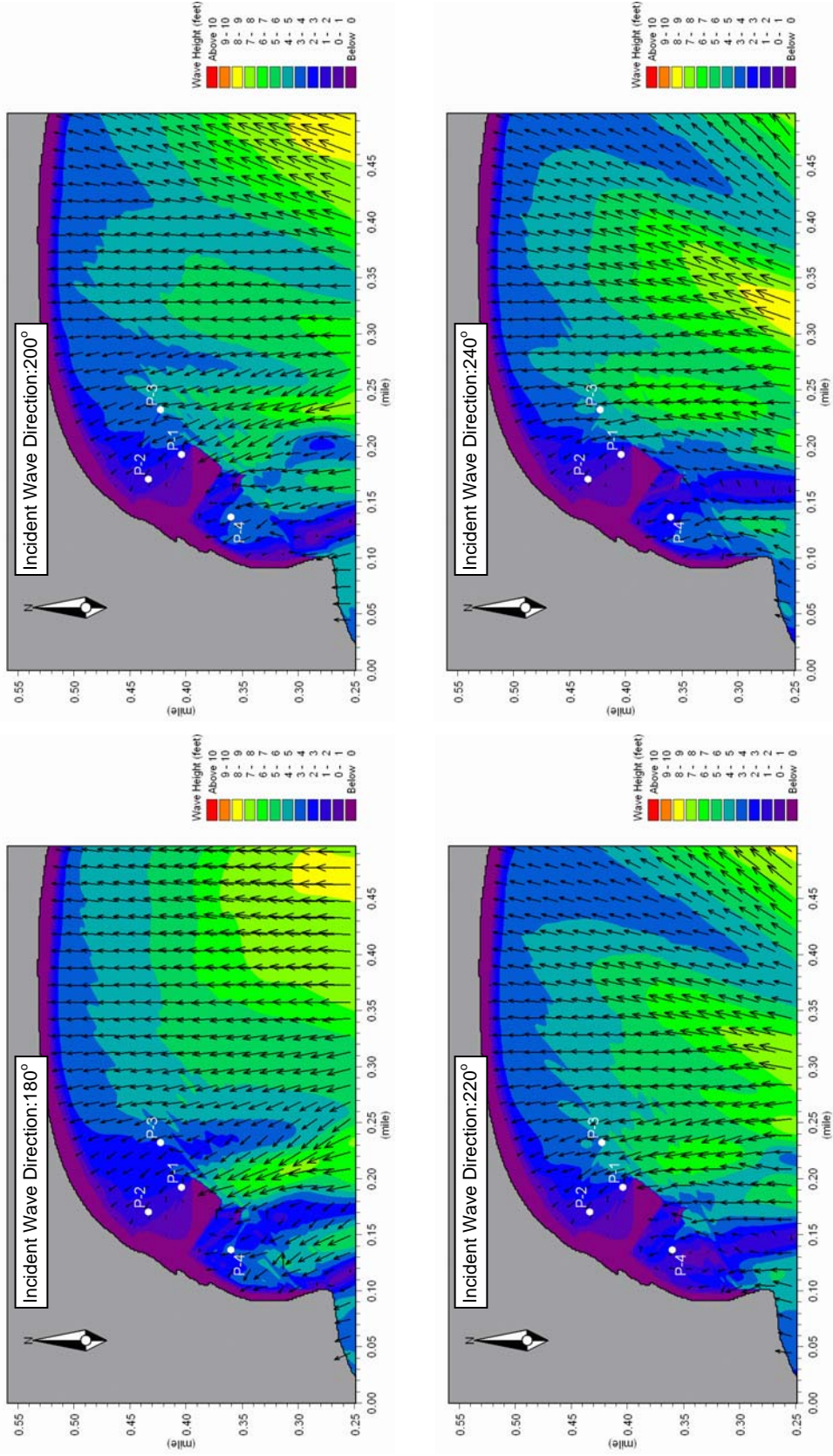


Figure 2.4-5: Nearshore Wave Transformation – Alternative 3 (7.5 ft @ 12 sec, MHW)

## 2.5 IMPROVEMENTS TO LAUNCH OPERATIONS

The existing breakwater at Shelter Cove was constructed many years ago and served well to create a harbor for boaters to launch and retrieve their boats. The existing breakwater was oriented so that it protects the harbor from both northwest and south swells. A concrete launch ramp was constructed to facilitate boat launching. Boats can be self launched by backing the boat down to the water via the concrete ramp or carried by high clearance tractors operated by the local boat launching business so that it can drive further into the deeper water for bigger boats. Sand was accumulating behind the breakwater and the harbor became shallower. At high tide, boats can usually be launched at station P-2 (see Figure 2.4-2), which is close to the concrete ramp. At low tide, boats have to be launched from further out, usually at P-1 which is closer to the end of the breakwater.

Due to large storm wave action, the existing breakwater has been damaged and the larger rocks have fallen off the breakwater, resulting in the lowering of the crest. As a result, it no longer provides as good a protection as it used to for boaters to launch and retrieve their boats at the harbor. When there are over 5 foot swells riding on high tide, the breakwater is submerged and waves overtop the breakwater putting the boaters at risk.

In order to restore the harbor to a safer place for launching, the breakwater has to be rehabilitated. As described above, two improvement alternatives were analyzed in this study. The effectiveness of the proposed breakwater rehabilitation on launch operations was analyzed by comparing the calculated wave height transmission coefficient for the breakwater for alternative conditions with those for the existing conditions. Here specifically, the wave height transmission coefficient for the breakwater was defined as the ratio of the wave height on the lee side of the breakwater to the incident wave height in 30-ft water depth. Waves in the lee side of the breakwater come from two sources: diffraction around the tip of the breakwater and overtopping over the crest of the breakwater. While the diffracted wave was obtained from the nearshore wave model, the transmitted wave due to overtopping was calculated based on the van der Meer and d'Angremond formula [Table VI-5-15 of Reference 3]. The total transmitted wave was calculated using the following formula:

$$C_t = \sqrt{C_{tD}^2 + C_{tO}^2}$$

where,

$C_t$  is the total wave transmission coefficient

$C_{tD}$  is the wave transmission coefficient due to diffraction

$C_{tO}$  is the wave transmission coefficient due to overtopping

The calculated total wave transmission coefficients for the Existing, Alternative 1, and Alternative 3 conditions are summarized in Table 2.5.1. From the table it can be seen that at MLW, the wave transmission coefficients for Alternative 1 are the same as those for Existing Conditions for all incident waves. For Alternative 3, the wave transmission coefficients are the same for those waves coming from 180° and 260°. For waves coming from 200°, 220° and 240°, there is an increase in wave transmission of up to

0.04 (or 4%), which is considered minor. For MHW conditions, wave transmission for both Alternatives 1 and 3 is reduced significantly. The maximum reduction is about 0.23 (or 23%). Therefore boats can launch and be retrieved more efficiently at high tide.

In summary, both Alternatives 1 and 3 result in reduced wave conditions for launch operations at high tide. The overall reduction in wave heights at Station P-2, at high tide, is slightly better for Alternative 1 than Alternative 3. Farther inside the harbor basin, the wave heights are the same for both Alternatives. Alternative 3 provides a larger basin for queuing and berthing of boats. The existing breakwater already protects the harbor well at low tide, therefore the results do not indicate any improvements in breakwater performance at low tide.

Table 2.5.1: Wave Transmission Into Harbor at MLW (P-1) and at MHW (P-2)

Wave Conditions @ 30-ft water depth		Wave Transmission Coefficient ( $C_t$ ) for:					
		Existing Condition		Alternative 1		Alternative 3	
Direction	Hs(ft), Tp(s)	MLW	MHW	MLW	MHW	MLW	MHW
180°	3.3ft, 9s	0.07	0.36	0.07	0.18	0.07	0.20
	7.5ft, 12s	0.09	0.31	0.09	0.19	0.11	0.21
200°	3.3ft, 9s	0.17	0.42	0.17	0.29	0.21	0.32
	7.5ft, 12s	0.10	0.31	0.10	0.19	0.11	0.21
220°	3.3ft, 9s	0.21	0.43	0.21	0.30	0.25	0.34
	7.5ft, 12s	0.10	0.31	0.10	0.19	0.11	0.21
240°	3.3ft, 9s	0.21	0.38	0.21	0.23	0.24	0.23
	7.5ft, 12s	0.10	0.29	0.10	0.16	0.11	0.16
260°	3.3ft, 9s	0.00	0.32	0.00	0.09	0.00	0.09
	7.5ft, 12s	0.00	0.26	0.00	0.09	0.00	0.09

## 2.6 EFFECT ON SURF CONDITIONS NEAR WASH ROCK

There were concerns about the potential impact of the breakwater rehabilitation on surf conditions near Wash Rock, which is located about 180 feet northeast from the tip of the existing breakwater. Since the reef near Wash Rock is generally sheltered from northwest swells by Pt. Delgada, waves in this area are gentle and predictable. It is used by children as well as seniors to learn and enjoy surfing. Therefore it is important to assure that the breakwater rehabilitation does not alter the existing surf in this area. Wave height and direction were used to describe surf conditions in the numerical model.

The effect on waves near Wash Rock was evaluated by looking at transformed wave height and direction at Wash Rock (station P-3 in Figure 2.4-2) for all possible incident waves. The wave transformation coefficients and directions at P-3 were obtained from the nearshore wave model results for Existing as well as Alternatives 1 and 3 conditions. Two typical water surface elevations, MLW and MHW were used. The results are summarized in Table 2.6.1 and Table 2.6.2 with the largest changes in bold typeface, and are discussed in the following text. In general, the simulated wave transformation coefficients and directions for Alternatives 1 and 3 are very close to that for Existing Conditions, regardless of incident wave height, period, direction or water level.

### Mean Low Water Conditions

For MLW conditions, there is no change in wave conditions for either alternative. This is to be expected because the breakwater is already effective during low to mid tide conditions. The values for the transformation coefficient ( $C_t$ ) and direction presented in Table 2.6.1 below are identical for all 3 cases.

Table 2.6.1: Wave Transformation Near Wash Rock (P-3), at MLW

Incident Wave @ 30-ft water depth		Wave Height Transformation Coefficients, $C_t$ and Direction at P-3					
		Existing Condition		Alternative 1		Alternative 3	
Direction	Hs(ft) & Tp (s)	$C_t$ at MLW	Dir at MLW	$C_t$ at MLW	Dir at MLW	$C_t$ at MLW	Dir at MLW
180°	3.3ft, 9s	0.70	168	0.70	168	0.71	168
	7.5ft, 12s	0.39	176	0.39	176	0.38	175
200°	3.3ft, 9s	0.57	170	0.57	170	0.57	171
	7.5ft, 12s	0.35	169	0.35	169	0.34	168
220°	3.3ft, 9s	0.78	171	0.78	171	0.78	172
	7.5ft, 12s	0.40	180	0.40	180	0.40	180
240°	3.3ft, 9s	0.79	175	0.79	175	0.79	174
	7.5ft, 12s	0.40	181	0.40	181	0.40	181
260°	3.3ft, 9s	0.25	185	0.25	185	0.26	186
	7.5ft, 12s	0.17	168	0.17	168	0.17	167

### Mean High Water Conditions

For MHW conditions, Alternative 1 does not alter wave conditions at Wash Rock. The largest simulated change is for a 240° direction, which shows a 0.04 change in  $C_t$ .

(corresponds to less than a 2-inch change in wave height), and a 3° change in direction. These are not significant numbers.

Although the results show some changes for Alternative 3 compared to Alternative 1, the effect is not considered significant for coastal processes. For example, the maximum change in  $C_t$  is 0.09 for waves from 220° which corresponds to about a 3-inch change in wave height. However, waves from a 220° wave approach direction have an occurrence of less than 4% (see Table 2.7.1), and is therefore considered not significant. Similarly, the maximum change in wave direction is 12° for waves from a 180° direction, which again has an occurrence of less than 4% of the time.

Table 2.6.2: Wave Transformation Near Wash Rock (P-3), at MHW

Incident Wave @ 30-ft water depth		Wave Height Transformation Coefficients, $C_t$ and Direction at P-3					
		Existing Condition		Alternative 1		Alternative 3	
Direction	Hs(ft) & Tp (s)	$C_t$ at MHW	Dir at MHW	$C_t$ at MHW	Dir at MHW	$C_t$ at MHW	Dir at MHW
180°	3.3ft, 9s	0.52	165	0.51	<b>162</b>	0.49	<b>153</b>
	7.5ft, 12s	0.44	160	0.44	160	0.47	167
200°	3.3ft, 9s	1.02	176	1.02	175	1.05	177
	7.5ft, 12s	0.58	168	0.59	168	0.62	174
220°	3.3ft, 9s	1.07	174	1.09	175	<b>1.16</b>	180
	7.5ft, 12s	0.61	169	0.62	171	0.63	172
240°	3.3ft, 9s	1.19	181	<b>1.23</b>	<b>184</b>	1.22	184
	7.5ft, 12s	0.66	182	0.64	181	0.61	178
260°	3.3ft, 9s	0.19	199	0.21	200	0.20	198
	7.5ft, 12s*	-	-	-	-	-	-

\* Not Simulated - Occurrence less than 0.5% from this direction (see Table 2.7.1)

## 2.7 EFFECT ON BEACH AND BLUFF EROSION

### 2.7.1 Morphology of Shelter Cove

Shelter Cove is a typical hook-shaped bay. This is a very common shape for beaches between two headlands, groins, or other fixed points along the Western US; it is also commonly bounded by areas with erosional bluffs. The shape is variously termed log-spiral, crenulate, or hook-shaped (e.g., References 4 and 5). Point Delgada acts as the upcoast (western) limit for hook-shaped bay that includes Shelter Cove; the rocky outcroppings approximately one mile to the southeast acts as the downcoast (eastern) limit.

The characteristic curved shape of hook-shaped bays arises from the wave conditions at the bay. Consider the arrival of uniform swell waves to a pocket beach – for example, the prevailing northwest waves in the area. The headland on the upcoast side of a hook-shaped bay, which is generally to the north or west in California, blocks waves approaching from the upcoast direction and acts as a diffraction point. This headland is always the most prominent. Diffraction of waves around the headland and refraction of waves as they approach the beach cause the waves crests to curve near this end, giving rise to the curved segment of the shoreline. The downcoast (south or east) structure anchors the other end of the bay; the shoreline is relatively straight near this end.

The aerial photograph in Figure 2.7-1 illustrates this pattern of diffraction and refraction around Point Delgada. The waves are actually diffracted twice: once around Point Delgada, and once around the tip of the breakwater. The swell offshore is from approximately northwest; the wave crests, visible on the left side of this photograph, are essentially straight. As the waves travel around Point Delgada, they turn towards the north and the wave crests become more curved. By the time the waves reach the shoreline in Shelter Cove, the wave crests are curved in an approximate log-spiral shape, and are essentially shore-parallel.



Figure 2.7-1: Wave Diffraction and Refraction around Point Delgada (1981 photograph)

The pattern of wave transformation from offshore to nearshore is also demonstrated on Figure 2.7-2. The figure shows wave roses in different water depths, based on results of the numerical modeling.

### 2.7.2 Sediment Transport Processes at Shelter Cove

Wherever wave crests strike a shoreline at an angle, they produce a longshore transport of beach sediment. Sediment continues to be shifted alongshore as long as the waves continue to break obliquely to the shore. The equilibrium shape of a shoreline is that for which the breakers are parallel to the beach everywhere along the shoreline – only then is there no longshore transport of sediment. In addition to longshore transport, there is a seasonal transport process called cross-shore transport, which erodes beaches in winter as sand is deposited offshore, and brings back beaches in summer as the offshore sand deposits come back. A beach in *equilibrium* implies seasonal variation in beach width as cross-shore and longshore transport processes occur.

As described above, the Shelter Cove littoral cell is bounded between Pt. Delgada (upcoast control) and the rocky bluff south of the Dead Man’s Gulch (downcoast control). There is little sediment coming in and out of the system along the coast (although cross-shore processes continue), and the shoreline change is largely dependent on local processes.

For the study area, a distribution of wave conditions is shown on Table 2.7.1, which gives a summary of the wave height and direction distribution in 30 ft water depth ½ mile south of the breakwater. The table was developed based on wave rose data at the buoy, and numerical simulations of offshore wave transformation.

Table 2.7.1: Wave Height and Direction in 30 ft Water Depth ½ Mile South of the Breakwater (from wave rose at the buoy and offshore wave simulations)

Wave Height (ft)	Direction (from true north)				
	180°	200°	220°	240°	260°
>13.2	0.18%	0.20%	0.27%	3.33%	-
9.9-13.2	0.77%	0.60%	0.32%	3.33%	-
6.6-9.9	0.71%	0.68%	0.56%	7.02%	0.12%
3.3-6.6	1.47%	5.36%	1.62%	12.54%	4.64%
0-3.3	0.32%	2.76%	0.99%	17.76%	34.45%
Total	3.45%	9.60%	3.76%	43.98%	39.21%

From the same wave data, the distribution near the breakwater is shown on Table 2.7.2, which implies that the incident wave direction is limited to a 160° to 200° band. As the wave approaches the shoreline, the direction changes further such that it induces a northwesterly sand transport (longshore transport). This is the source of sand within the basin in Shelter Cove, as well as for the beach where the breakwater connects to the shoreline. The shore-normal leg of the breakwater interrupts this supply to reaches farther west, upto the Point. South storms in particular drive this sediment into the harbor, where it would normally be deposited onto the beach and move in a onshore-offshore direction as cross-shore transport. The offshore leg of the breakwater does not allow this to happen, and the sand remains within the harbor basin. Occasional

dredging in the harbor has happened over time, and sand taken from behind the breakwater.

Because the breakwater has settled and is overtopped at high tide, there is some sediment transport over the breakwater which has resulted in a small pocket beach west of the breakwater, between the breakwater and the rocky point. The beach does not extend far beyond where the breakwater connects to shore. Although the amount transported over and through the breakwater is small – the pocket beach is not completely isolated from the littoral system of Shelter Cove.

Table 2.7.2: Wave Direction Distribution near the Breakwater Tip

Direction	% Occurrence
160°	3%
170°	12%
180°	45%
190°	20%
200°	20%

If the breakwater rehabilitation project envisioned interrupting this process by constructing shore normal structures, the processes would change. However, the breakwater rehabilitation for Alternatives 1 and 3 will only raise the portion of breakwater beyond the beach. The elevation of the shore connected portion of the inner breakwater leg will remain the same for both concepts so that the long shore sediment transport flow will not be interrupted. The small amount of transport which is leaking out of the basin through and over the existing low crested offshore portion of the breakwater will be interrupted, and will accumulate within the harbor. The sand presently leaking out of the basin is not accumulating outside the basin – the presence of tide pools demonstrates this.

For assessing beach and bluff erosion west of the breakwater, wave conditions in the area (station P-4) were compared. Wave transformation coefficients and directions at P-4 were extracted from the nearshore wave model, and results summarized in Table 2.7.3 and Table 2.7.4.

Table 2.7.3: Transformation Coefficients (Ct) And Wave Directions (Dir) At P-4 (MLW)

Incident Wave		Existing Condition		Alternative 1		Alternative 3	
Direction	Hs(ft) & Tp (s)	Ct at MLW	Dir at MLW	Ct at MLW	Dir at MLW	Ct at MLW	Dir at MLW
180°	3.3ft, 9s	0.13	158	0.13	158	0.13	158
	7.5ft, 12s	0.05	171	0.05	171	0.05	171
200°	3.3ft, 9s	0.14	155	0.14	155	0.14	155
	7.5ft, 12s	0.07	157	0.07	157	0.07	157
220°	3.3ft, 9s	0.07	160	0.07	160	0.07	160
	7.5ft, 12s	0.07	158	0.07	158	0.07	158
240°	3.3ft, 9s	0.12	159	0.12	159	0.12	159
	7.5ft, 12s	0.04	151	0.04	151	0.04	151
260°	3.3ft, 9s	0.02	165	0.02	165	0.02	165
	7.5ft, 12s	0.01	178	0.01	178	0.01	178

Table 2.7.4: Transformation Coefficients (Ct) And Wave Directions (Dir) At P-4 (MHW)

Incident Wave		Existing Condition		Alternative 1		Alternative 3	
Direction	Hs(ft) & Tp (s)	Ct at MHW	Dir at MHW	Ct at MHW	Dir at MHW	Ct at MHW	Dir at MHW
180°	3.3ft, 9s	0.06	148	0.06	147	0.06	147
	7.5ft, 12s	0.12	169	0.12	170	0.12	170
200°	3.3ft, 9s	0.21	152	0.21	152	0.21	152
	7.5ft, 12s	0.13	171	0.13	171	0.13	171
220°	3.3ft, 9s	0.10	179	0.10	179	0.10	179
	7.5ft, 12s	0.06	177	0.06	177	0.06	177
240°	3.3ft, 9s	0.28	168	0.28	168	0.28	168
	7.5ft, 12s	0.11	164	0.11	164	0.11	164
260°	3.3ft, 9s	0.03	193	0.03	193	0.03	193
	7.5ft, 12s	0.02	166	0.02	166	0.02	166

Based on the simulation results it can be concluded that there is no change in wave conditions due to breakwater rehabilitation in the area between Point Delgada and the breakwater. Therefore no adverse effect is expected on the beach and bluff west of the breakwater. Similarly, no changes are anticipated to the beaches east of the launch ramp, because littoral transport processes will not change there.

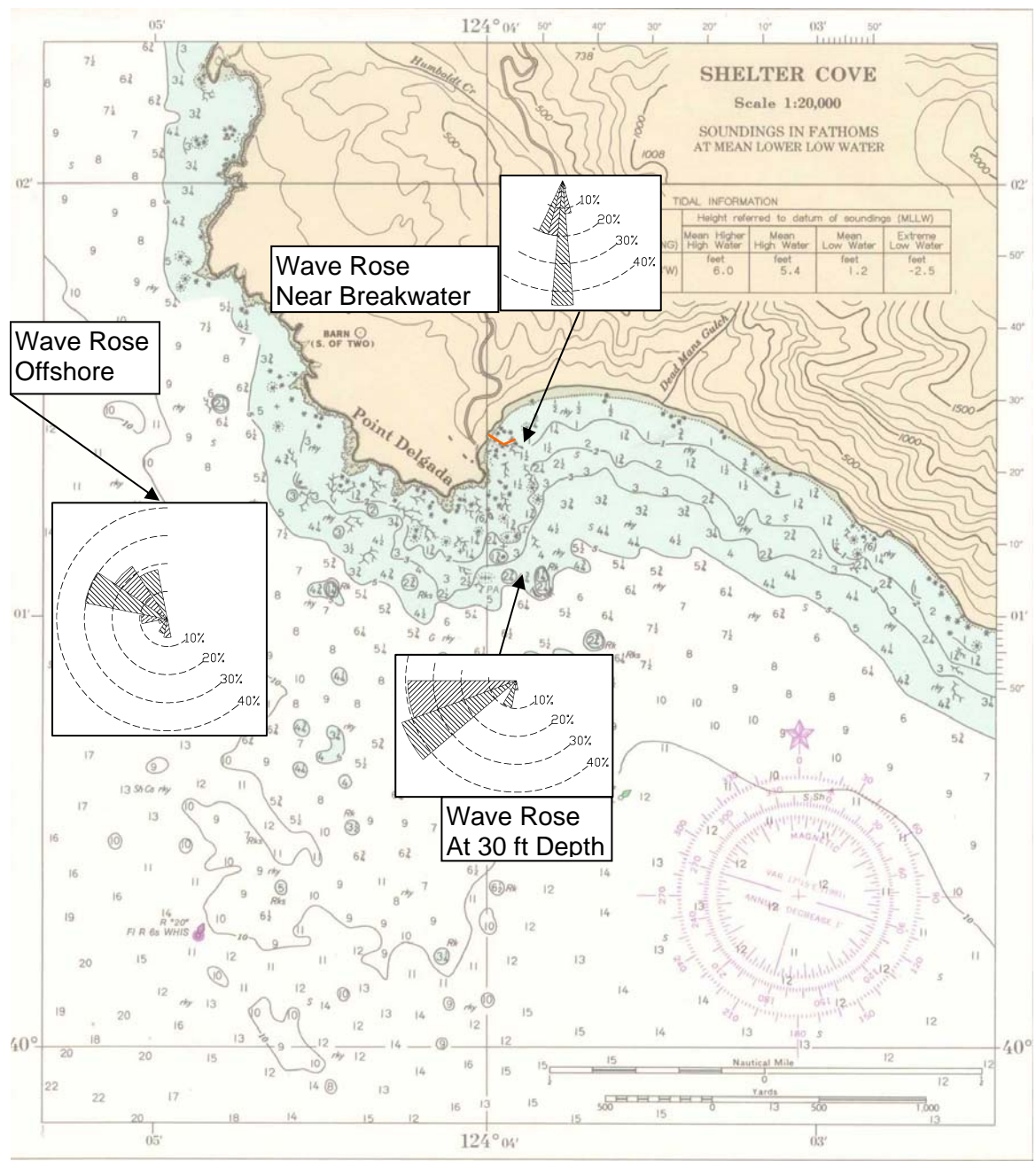


Figure 2.7-2: Wave Transformation from Offshore to Nearshore

### 3. MAINTENANCE & CONSTRUCTION ISSUES

#### 3.1 OPERATION & MAINTENANCE ISSUES

##### 3.1.1 Breakwater Maintenance Repair

A design wave height of 10 feet was used for determining the breakwater armor protection, which is considered to be a practical extreme wave height. The rehabilitated breakwater may need maintenance as a result of extreme El Nino type of storm wave + surge conditions.

It is assumed that a 5%-10% damage of the armor layer will occur on average every 10 years. Assuming a unit cost of \$80 per ton for armor rock repair and 25% contingency, the repair cost for each alternative is shown in Table 3.1-1. We recommend an average annual maintenance budget of about \$3,000 be set aside for breakwater repairs.

Table 3.1-1: Probable Breakwater Maintenance Cost

Parameter	Alternative 1		Alternative 3	
	5% Damage	10% Damage	5% Damage	10% Damage
Maintenance Requirement (5-10 Ton Armor Rock)	110 tons	220 tons	160 tons	320 tons
Armor Rock Repair Cost	\$8,800	\$17,600	\$12,800	\$25,600
Contingency (25%)	\$2,200	\$4,400	\$3,200	\$6,400
Total Maintenance Cost	\$11,000	\$22,000	\$16,000	\$32,000

##### 3.1.2 Harbor Maintenance Dredging

Sediment transport is a function of the bed shear stresses imposed by the waves on the bottom. The simulations indicate that for existing condition the waves are reduced to about 35% on average compared to incident waves at 30-ft depth. For Alternative conditions the waves are reduced to about 20% on average compared to incident waves at 30-ft depth (see Table 2.5-1). Standard relationships between wave height, bed shear stresses and potential for depositions indicate that there would be a two-fold increase in potential for deposition for alternative conditions within the harbor resulting from the rehabilitation.

Estimating the true longshore transport would typically require conducting transport calculations for each wave height, period, direction and grain size distribution and performing statistics based on relative rates of occurrence. For this study, recognizing that there is no formal maintenance dredging program for the harbor, we anticipate that there will be a need for maintenance dredging for either of the alternatives. For budgeting purposes, using the results of the simulations and engineering judgment, we recommend that an allowance for dredging about 2000 cubic yards every five years be set aside. This corresponds to less than 6 inches of deposition per year within the deeper part of the basin. This would occur after storms with a southerly component. Using a unit rate of \$20 per cubic yard, we recommend that an average annual maintenance budget of about \$8,000 be set aside for maintenance dredging.

### 3.2 CONSTRUCTION ISSUES

The breakwater rehabilitation will take place from land using dozers and other tire/track equipment. Crushed rock will be used to construct an accessible path along the existing crest of the breakwater, and placement will start at the seaward tip of the structure, moving landward. The proposed 11-foot crest width will be wide enough for this. The smaller rocks that have sloughed off the breakwater could potentially be used in place of imported crushed rock for this purpose.

All new rock used in the rehabilitation would be sourced from local quarries, and transported by trucks along Shelter Cove Road. The amount of rock for each Alternative is as follows :

<u>Quantity</u>	<u>Unit</u>	<u>Alternative 1</u>	<u>Alternative 3</u>
3 - 5 ton armor rock	tons	4,240	6,360
500 - 1000 lb rock	tons	650	1,920
6-inch rock	tons	300	1,350
Total imported rock	tons	5,190	9,360

Using a 25-ton capacity for trucks (about 15-17 CY/truck), Alternative 1 implies about 200 round trips between the quarry and the launch ramp. Alternative 3 implies about 375 round trips. The environmental review document for this project should evaluate whether this is an impact of significance.

It is expected that construction for Alternative 1 would last just about 1 month including mobilization and weather delays. Of this, delivery of rock would most likely be about 10-15 days. Alternative 3, if selected, would take about 2.5 to 3 months, with delivery of rock taking place over about 20 days. A staging area would be needed near the top of the bluff to stockpile some of the rock and to store equipment over the construction period.

The construction will have to be timed during periods of low tide and offshore swell, as well as consider the fishing season, when ramp usage is at a maximum.

## 4. SUMMARY

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This study was conducted to address comments received on the Initial Study, with the primary objectives being assessing the effects of rehabilitation on launch operations, surf conditions, and beach/bluff stability for Alternatives 1 and 3. The analysis included numerical simulations for the range of incident wave conditions (height and period) that the breakwater is exposed to, and an assessment of wave transformation parameters at key locations that represent the effects of the breakwater. A summary of the analysis is presented below.

1. Immediately offshore of the Shelter Cove launch facility, the full direction band of the offshore waves (northwest through south directions) is essentially reduced to a southwest quadrant incident wave approach direction. Therefore, for nearshore wave simulation, it is only necessary to simulate waves from this quadrant to represent the wider offshore wave direction band. Results are described in Section 2.3.
2. Alternatives 1 and 3 both result in a reduction in wave energy at high tide in the lee of the breakwater, which will benefit launching/retrieval of boats. Immediately behind the breakwater, the overall reduction in wave heights is slightly better for Alternative 1 than for Alternative 3, primarily because of the alignment of the breakwater. However, Alternative 3 provides a larger basin (about 10,000 ft<sup>2</sup> larger) for queuing and berthing of boats. Farther inside the harbor basin, the wave heights are the same for both Alternatives. The existing breakwater already protects the harbor well at low tide, therefore the results do not indicate any improvements in breakwater performance at low tide. Results are described in Sections 2.4 and 2.5.
3. The analyses for effects at Wash Rock indicate that, in general, wave transformation coefficients and directions for Alternatives 1 and 3 are very close to that for Existing Conditions, regardless of incident wave height, period, direction or water level. Alternative 1 does not alter wave conditions at Wash Rock (simulations for both MLW and MHW conditions indicate no change). Alternative 3 simulations also show no change in wave conditions at Wash Rock for low to mid-tide conditions. For MHW conditions, Alternative 3 shows a 0.04 change in  $C_r$  (corresponds to less than a 2-inch change in wave height), and a 3° change in direction, which are not significant numbers. Results are described in Section 2.6.
4. The breakwater rehabilitation concept for both alternatives envision raising the portion of breakwater beyond the beach. The elevation of the shore connected portion of the inner breakwater leg will remain the same for either alternative, so that the long shore sediment transport flow will not be interrupted. As a result, the simulations indicate no change in wave conditions due to breakwater rehabilitation in the area between Point Delgada and the breakwater. Therefore no adverse effect is expected on the beach and bluff west of the breakwater. Similarly, no changes are anticipated to the beaches east of the launch ramp, because littoral transport processes will not change there. Results are described in Section 2.7.
5. Alternative 3 has a higher capital cost because of the larger structure, as well as a higher operations and maintenance cost associated with repairs and dredging (see Section 3.1). However, it provides a larger basin area for queuing of boats.

6. Since Alternative 3 envisions a larger construction operation and a longer construction duration, traffic and noise effects need to be evaluated. In addition, since the footprint of the structure is larger than at present our experience suggests that it will be subject to greater scrutiny by regulatory and resource agencies, as well as environmental groups, because it will be perceived as an *Improvement Project* rather than a *Rehabilitation Project*. Also, the benefits of a larger basin (increase in number of boats being able to launch or be retrieved) will need to be evaluated against the cost of implementing this alternative.

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